Design one way ribbed floor system shown below subjected to partition load of 3kN/m and live load of 5kN/m2 materials used include C-30, S-300 and class I work. Ribbing in the X –direction

5m

6m

7m

Solution

Cross section of ribbed slab

$$b\_{w}\geq 70mm say b\_{w}=80mm $$

$$d\_{w}\geq 4\*70mm=280mm $$

Ribbed spacing
$$S\leq 1000mm for practical case S=400mm $$

Clear distance
$$b\_{e}=400-80=320mm $$

Slab thickness hf

$$d\_{f}=50mm or \frac{1}{10}\*clear distance\left(b\_{e}\right)=\frac{1}{10}\*320=32mm $$

Take the larger of the two df= 60mm For practical case $d\_{w}\leq 280$mm and D=300mm
$$b\_{w}+d\_{f}=300mm \gg \gg d\_{w}=240mm$$

60mm

400

80mm

The rib is to be considered like a T section

be

hf

dw

bw

Considering Second panel S2 (slab2)

$$b\_{e}\leq b\_{w}+\frac{L\_{e}}{5}=80+\frac{5000}{5}=1080mm or C/C b/n beam spacing =400mm$$

Therefore take $b\_{e}=400mm$

Loading

Dead load, self weight =$25\left(0.08\*0.24+0.06\*0.4\right)=1.08kN/m$

Floor finishing (cement screed +PVC tile) $(23\*0.03+0.02\*16)\*0.4=0.404kN/m$

Partition load (DL) =$3\*0.4=1.2kN/m$

Total dead load (DL) = 1.08+0.404 $1.2=2.684kN/m$

Live load LL $=5\*0.4=2kN/m$

Design load $(P\_{d}=1.3DL+1.6LL=1.3\*2.684+1.6\*2=6.69kN/m$

**Analysis**

Take into account the variation of load

1. Maximum support moment (for maximum loading case

24.4

5m

6m

3EI/L

3EI/L

 Rel. K

6

5

 DF

0.55

0.45

-30.11

 BM

20.91

+4.14

+5.06

25.97

25.97

24.4

15.74

11.53

21.92

15.74

25.77

11.53

18.52

9.44

1. Maximum span moment for (DL only on second panel)

2.68

6m

10

1.29

20.82

20.33

16.68

2.63

11.53

23.46

10.77

16.68

20.33

-20.33

+11.95

+9.78

8.38

-30.11

0.55

0.45

6

3EI/L

3EI/L

5m

23.46

 Rel k

 FEM

10

2.68

SFD

2.63

BMD

Design constants

$$C-30 \gg \gg f\_{ck}=\frac{30}{1.25}=24MPa$$

$$f\_{cd}=0.85\*\frac{f\_{ck}}{1.5}=\frac{0.85\*24}{1.5}=13.6MPa$$

S-300 $\gg \gg f\_{yk}=400, f\_{yd}=\frac{f\_{yk}}{1.15}=\frac{300}{1.15}=260.87MPa $

$$m=\frac{f\_{yd}}{0.8f\_{ck}}=\frac{260.87}{0.8\*13.6}=23.98$$

$c\_{2}=0.32f\_{cd}=0.32\*13.6=4.352 $and c1 =2.5

400mm

60mm

240mm

80mm

For Maximum Span Moment, Mmax =20.82kN.m assuming b=be=400mm

$$k\_{x}=0.5\*(c\_{1}-\sqrt{c\_{1}^{2}-\frac{4M\_{d}}{bd^{2}c\_{2}}}=0.5\*(2.5-\sqrt{2.5^{2}-\frac{4\*20.82}{300\*540^{2}\*3.616}}=0.073<k\_{x max} $$

Amount of reinforcement

$$A\_{s}=\frac{k\_{x}}{m}bd=\frac{0.073}{23.98}\*400\*259=315.38mm^{2}$$

$$ρ\_{min}=\frac{0.5}{f\_{yk}}=\frac{0.5}{400}=0.00125\gg \gg \gg A\_{s min}=ρ\_{min}bd=0.00125\*400\*259=129.5mm^{2} $$

$$A\_{s}>A\_{Smin} ok$$

$$Let use ∅12 a\_{s}=\frac{d^{2}π}{4}=π\*\frac{12^{2}}{4}=113.1mm^{2}$$

$$number of bars ∅12=\frac{315.38}{113.1}=2.79 \gg \gg use 3∅12 bars $$

For maximum support moment Mmax =25.97kN.m

For the negative bending moment consider rectangular beam b=bw =80mm check for d value.

$$d=\sqrt{\frac{M\_{max}}{0.8f\_{cd}bk\_{x max}(1-0.4k\_{x max})}}=\sqrt{\frac{25.97\*10^{6}}{0.8\*13.6\*80\*0.448\*\left(1-0.4\*0.448\right)}}=284.85mm≅285mm>259mm not safe$$

Increase the value of bw =100mm

$$d=\sqrt{\frac{M\_{max}}{0.8f\_{cd}bk\_{x max}(1-0.4k\_{x max})}}=\sqrt{\frac{25.97\*10^{6}}{0.8\*13.6\*100\*0.448\*\left(1-0.4\*0.448\right)}} =258mm<259mm safe$$

Use bw =100mm the load increase =25(0.24\*0.02) =0.12kN/m

$$P\_{d}=1.3\*0.12=0.156kN/m$$

Moment incremental =
$$M\_{d}=\frac{6.69+0.156}{6.69}\*by initial moment =1.02\*25.97=26.58kN.m$$

$$k\_{x}=0.5\*(c\_{1}-\sqrt{c\_{1}^{2}-\frac{4M\_{d}}{bd^{2}c\_{2}}}=0.5\*(2.5-\sqrt{2.5^{2}-\frac{4\*26.58}{100\*259^{2}\*4.382}}=0.439<k\_{x max} $$

Amount of reinforcement

$$A\_{s}=\frac{k\_{x}}{m}bd=\frac{0.439}{23.98}\*100\*259=473.79mm^{2}$$

$$Let use ∅12 a\_{s}=\frac{d^{2}π}{4}=π\*\frac{12^{2}}{4}=113.1mm^{2}$$

$$number of bars ∅12=\frac{473.79}{113.1}=4.19 \gg \gg use 5∅12 bars $$

**Shear reinforcement**

In the vicinity of the support assume supporting beam with to be 300mm. the largest shear at d-distance from face of support (in the case of loading case 1)

Shear capacity of the section

$$V\_{C}=0.25f\_{ckd}k\_{1}k\_{2}b\_{w}d$$

$$f\_{ctd}=\frac{0.21(f\_{ck})^{2/3}}{γ\_{c}}=\frac{0.21\*(24)^{2/3}}{1.5}=1.165MPa$$

$$k\_{1}=1+50ρ, \gg \gg \gg ρ=\frac{A\_{s}}{bd}=5\*\frac{113}{100\*259}=0.0218$$

$$k\_{1}=1+50\*0.0218=2.09>2 thus take k\_{1}=2 and k\_{2}=1.6-d=1.6-0.259=1.341$$

$$V\_{C}=0.25f\_{ckd}k\_{1}k\_{2}b\_{w}d=0.25\*1.165\*2\*1.341\*100\*259\*10^{-3}=20.23kN≅V\_{sd}$$

Since
$V\_{C}≅V\_{sd}$ provide minimum reinforcement

$$S\_{max}=\frac{A\_{v}f\_{yk}}{0.4b\_{w}}=\frac{56\*300}{0.4\*100}=420mm$$

 smax = 0.5d =0.5\*259=129.5mm ≤ 300mm if Vsd ≤ 

Use Φ8 stirrups c/c 130mm

**The transverse reinforcement at topping cross sectional area per rib**

$A\_{C}=60\*400+240\*100=48000mm^{2}$

Based on EBCS -2 $A\_{st}=0.001A\_{C}=0.001\*48000=48mm^{2}$

Spacing $∅8=\frac{400\*50}{60}=333.33 use ∅8\frac{c}{c}330mm<400mm ok$

**Longitudinal shear (υsd) per unit length**

**Flange in compression**

Compression under span moment (Mmax) =20.82kN. m

$$k\_{x max}=0.448=\frac{x}{d}\gg \gg x=0.448d=0.448\*259=116.03mm$$

$$Z=d-0.4x=259-0.4\*116.03=212.59mm$$

$$b=400mm, b\_{w}=100mm$$

$$υ\_{sd}=\left(\frac{b\_{e}-b\_{w}}{2b\_{e}}\right)\*\frac{V\_{sd}}{Z}=\left(\frac{400-100}{2\*400}\right)\*\frac{20.63}{212.59}=36.39kN$$

$$V\_{Rd1}=0.25f\_{cd}h\_{f}=0.25\*13.6\*60=204kN>36.39kN \_{}$$

*Therefore it is safe against failure by crushing concrete*

$$V\_{Rd1}=0.5f\_{ctd}h\_{f}+\frac{A\_{sf}f\_{yd}}{S\_{f}} \gg \gg A\_{sf}=50\*\frac{1000}{330}=151.5mm^{2}$$

$$V\_{Rd1}=0.5\*1.165\*60+\frac{151.5\*260.87}{330}=154.713kN>36.39kN safe$$